## 3.2.1 LRFD Deck Design

A standard deck is defined as a deck slab on longitudinal beams with main reinforcement placed perpendicular to traffic.

As outlined in Article 9.6.1, the AASHTO LRFD Code permits three methods or procedures for designing bridge decks with primary reinforcement perpendicular to the main bridge beams. These are: (a) Approximate Elastic or “Strip” Method (4.6.2.1); (b) Empirical Design (9.7.2); and (c) Refined Analysis (4.6.3.2). The LRFD Deck Design Chart in Section 3.2.1 of the Bridge Manual was developed using the Strip or Approximate Elastic Method. This procedure is very similar to the slab design procedure in AASHTO LFD and thus provides a measure of continuity for engineers during the transition process from LFD to LRFD. Refined Analysis utilizes finite elements, which is unnecessary for standard deck design. Empirical Design employs the notion that the deck behaves more like a “membrane” than a series of continuous beams. While this may be true, it is not a well enough established design technique to be advocated by IDOT.

In the Approximate Elastic Method, the deck is designed for Flexural Resistance (5.7.3.2) and Control of Cracking (5.7.3.4). Limits of Reinforcement are also checked, but do not typically control in a standard deck design.

Shear design is not required for deck slabs (C4.6.2.1.6). Fatigue and Fracture design is also not required (9.5.3).

In a standard deck, three components are designed. Positive moment (bottom of slab transverse) reinforcement and negative moment (top of slab transverse) reinforcement are designed for the Approximate Elastic Method. Additional negative moment reinforcement for deck overhangs should also be designed with a significant Crash Loading normally governing. However, this is typically not required if the details provided in Section 3.2.4 of the Bridge Manual are followed for reinforcing the overhang and parapets. The 34 in. and 42 in. F-Shape parapets, in conjunction with the deck overhang designs, are rated for Crash Test Level TL-4 and TL-5, respectively, which is adequate for most situations.

Longitudinal reinforcement is not designed. The top longitudinal reinforcement need only satisfy Shrinkage and Temperature Requirements (5.10.8), where #5 bars at 12 in. centers are adequate. The bottom longitudinal reinforcement area is a percentage of the bottom transverse reinforcement (9.7.3.2). The percentage is 67% for all bridges with beam spacings within the limits of the standard deck design charts.

Additional longitudinal reinforcement is required for continuous span structures over the piers. See Sections 3.2.2 and 3.2.4 of the Bridge Manual for more information.

Reinforcement shall be developed to satisfy Section 5.11.1.2 of the LRFD Code. Extending the negative moment reinforcement to the end of slab and the positive moment reinforcement to one foot from the end of slab satisfies these requirements.

**LRFD Deck Slab Design Procedure, Equations, and Outline**

*Design Stresses*

$f'_c$ = 3.5 ksi
$f_y$ = 60 ksi

*Design Thickness*

The IDOT standard slab thickness is defined as 8 in. for all girder spacings between 5 ft. - 6 in. and 9 ft. - 6 in. This increase from the former standard of 7 ½ in. was specified to create more durable decks.

For girder spacings exceeding the boundaries mentioned above, the standard design charts are not applicable.

*Determine Maximum Factored Loading*

When designing deck slabs, two load combinations are used:

Strength I load combination, used in Flexural Resistance, is defined as:

$M_{STRENGTH\ I} = \gamma_p DC + \gamma_p DW + 1.75(LL + IM + CE)$ (Table 3.4.1-1)

Where $\gamma_p$ is equal to 1.25 (max.) for DC and 1.5 (max.) for DW.

Service I load combination, used in Control of Cracking, is defined as:

$M_{SERVICE\ I} = 1.0(DC + DW + LL + IM + CE)$ (Table 3.4.1-1)

The load abbreviations are defined as follows:

| | | |
|---|---|---|
| CE | = | vehicular centrifugal force, including forces due to bridge deck superelevation |
| DC | = | dead load of structural components (DC1) and non-structural attachments (DC2). Standard deck slabs are not designed for DC2 loading. |
| DW | = | dead load of future wearing surface |
| IM | = | dynamic load allowance (impact) |
| LL | = | vehicular live load |

Dead load (DC1 and DW) design moments are computed as $wL^2/10$. L is defined as the center-to-center beam spacing (4.6.2.1.6) for positive moment calculation. For negative moment, L is taken as that defined in Bridge Manual Figure 3.2.1-2.

Standard parapet, sidewalk, and railing loads are considered DC2 loading and are not used in the main reinforcement design. Bridges with large additional DC2 loads may require a non-standard deck design.

Live loads are taken from AASHTO LRFD Table A4-1. This table gives the Live Load Moment per ft. width for a given beam spacing. These values are already corrected for multiple presence factors and impact loading. Note that Bridge Manual Figure 3.2.1-2 defines span lengths for negative moment regions differently than positive moment regions. These span lengths fall within the limitations of AASHTO LRFD Section 4.6.2.1.6.

All factored loads shall then be multiplied by the load modifier $\eta_i$, defined as:

$\eta_i = \eta_D \eta_R \eta_I \geq 0.95$ (1.3.2.1-1)

Where:

$\eta_D$ = ductility factor, taken as 1.00 for conventional designs

$\eta_R$ = redundancy factor, taken as 1.00 for conventional levels of redundancy

$\eta_I$ = importance factor, taken as 1.00 for typical bridges

For most bridges, $\eta_i = (1.00)(1.00)(1.00) = 1.00$

*Check Flexural Resistance* (5.7.3.2)

The factored resistance, $M_r$ (k-in.), shall be taken as:

$$M_r = \phi M_n = \phi\left[A_s f_s\left(d_s - \frac{a}{2}\right)\right] \geq M_{STRENGTH\,1}$$ (Eqs. 5.7.3.2.1-1 & 5.7.3.2.2-1)

Where:

$\phi$ = Assumed to be 0.9, then checked in Limits of Reinforcement check

$a$ = depth of equivalent stress block (in.), taken as $a = c\beta_1$

$c$ = $\dfrac{A_s f_s}{0.85\beta_1 f'_c\, b}$ (in.) (Eqs. 5.7.3.1.1-4 or 5.7.3.1.2-4)

$A_s$ = area of tension reinforcement in strip (in.$^2$)

$b$ = width of design strip (in.)

$d_s$ = distance from extreme compression fiber to centroid of tensile reinforcement (in.)

$f_s$ = stress in the mild steel tension reinforcement as specified at nominal flexural resistance (ksi). As specified in Article 5.7.2.1, if $c / d_s < 0.6$, then $f_y$ may used in lieu of exact computation of $f_s$.

$f'_c$ = specified compressive strength of concrete (ksi)

$\beta_1$ = stress block factor specified in Article 5.7.2.2

$$\therefore\ M_r = \phi M_n = \phi\left[A_s f_s\left(d_s - \frac{1}{2}\frac{A_s f_s}{0.85f'_c\, b}\right)\right]$$

*Check Control of Cracking* (5.7.3.4)

The spacing of reinforcement, s (in.), in the layer closest to the tension face shall satisfy the following:

$$s \le \frac{700\gamma_e}{\beta_s f_s} - 2d_c \qquad (5.7.3.4\text{-}1)$$

Where:

$d_c$ = thickness of concrete cover from extreme tension fiber to center of the flexural reinforcement located closest thereto (in.). The 2005 interims of the AASHTO LRFD Code eliminated the two inch maximum clear cover value previously associated with this variable. Use the actual value, even if the clear cover is greater than two inches.

$\beta_s$ = $1 + \dfrac{d_c}{0.7(h - d_c)}$

h = slab depth (in.)

$f_s$ = stress in mild steel tension reinforcement at service load condition

= $\dfrac{M_{SERVICE\ I}}{A_s j d_s}$ (ksi)

j = $1 - \dfrac{k}{3}$

k = $\sqrt{(\rho n)^2 + 2\rho n} - \rho n$

$\rho$ = $\dfrac{A_s}{bd_s}$

n = $\dfrac{E_s}{E_c}$, typically taken as 9 for 3.5 ksi concrete (C6.10.1.1.1b)

$\gamma_e$ = 0.75 for Class 2 Exposure. C5.7.3.4 defines Class 2 Exposure as decks and any substructure units exposed to water

*Check Limits of Reinforcement* (5.7.3.3)

Check Maximum Reinforcement (5.7.3.3.1)

The 2006 Interims to the AASHTO LRFD Code do not explicitly state an absolute limit on the amount of reinforcement that can be used in a section. Rather, the code imposes reduced resistance factors for sections that experience very small amounts of strain i.e. are over-reinforced.

To determine whether or not a reduced resistance factor should be used, the tensile strain may be computed using the following equation:

$$\varepsilon_t = \frac{0.003(d_t - c)}{c} \qquad \text{(C5.7.2.1-1)}$$

Where:

$d_t$ = distance from extreme compression fiber to centroid of bottom row of reinforcement (in.) As there is typically only one row of reinforcement in slab bridges, $d_t = d_s$.

$$c = \frac{A_s f_s}{0.85 \beta_1 f'_c\, b} \qquad \text{(5.7.3.1.2-4)}$$

For $\varepsilon_t \geq 0.005$, the full value of $\phi = 0.9$ is used.

For $0.002 < \varepsilon_t < 0.005$, $\phi = 0.65 + 0.15\left(\frac{d_t}{c} - 1\right)$

For $\varepsilon_t \leq 0.002$, $\phi = 0.75$

The flexural resistance shall then be recalculated using this resistance factor, and a change in design made if necessary.

<u>Check Minimum Reinforcement</u> (5.7.3.3.2)

The minimum reinforcement shall be such that:

$M_r > 1.33M_{STRENGTH\ 1}$, or

$M_r > M_{cr}$

Where:

$M_{cr}$ = $\gamma_3\gamma_1 S f_r$ (k-in.) (Eq. 5.7.3.3.2-1)

$S$ = $\frac{1}{6}bh^2$ (in.$^3$)

$f_r$ = $0.24\sqrt{f'_c}$ (ksi) (5.4.2.6)

$\gamma_3$ = 0.75 for A706, Grade 60 reinforcement

$\gamma_1$ = 1.6 for non-segmentally constructed bridges

## LRFD Deck Slab Design Example: 7 ft. Beam Spacing, Positive Moment Reinforcement

### *Design Stresses*

$f_y$ = 60 ksi

$f'_c$ = 3.5 ksi

### *Design Thickness*

Standard eight inch slab thickness.

### *Determine Maximum Factored Loading*

<u>Unfactored Loads and Moments</u>

$$w_{DC1} = \left(\frac{0.150\ \text{k}}{\text{ft.}^3}\right)(0.667\ \text{ft.})(1\ \text{ft.}) = 0.100\,\frac{\text{k}}{\text{ft.}}$$

$$w_{DW} = \left(\frac{0.050\ \text{k}}{\text{ft.}^2}\right)(1\ \text{ft.}) = 0.050\,\frac{\text{k}}{\text{ft.}}$$

$$M_{DC1} = \frac{1}{10}\left(0.100\,\frac{\text{k}}{\text{ft.}}\right)(7\ \text{ft.})^2 = 0.490\ \text{k-ft.}$$

$$M_{DW} = \frac{1}{10}\left(0.050\,\frac{\text{k}}{\text{ft.}}\right)(7\ \text{ft.})^2 = 0.245\ \text{k-ft.}$$

$M_{LL+IM}$ = 5.21 k-ft. (Appendix A4)

<u>Factored Moments</u> (Table 3.4.1-1)

$$M_{STRENGTH\ I} = \eta_i[1.25M_{DC1} + 1.5M_{DW} + 1.75M_{LL+IM}]$$

$$= 1.00[1.25(0.490\ \text{k-ft.}) + 1.5(0.245\ \text{k-ft.}) + 1.75(5.21\ \text{k-ft.})]$$

$$= 10.10\ \text{k-ft}\left(\frac{12\ \text{in.}}{\text{ft.}}\right)$$

$$= 121.20\ \text{k-in.}$$

$$M_{SERVICE\ I} = \eta_i[1.0M_{DC1} + 1.0M_{DW} + 1.0M_{LL+IM}]$$

$$= 1.00[1.0(0.490\ \text{k-ft.}) + 1.0(0.245\ \text{k-ft.}) + 1.0(5.21\ \text{k-ft.})]$$

$$= 5.95\ \text{k-ft.}\left(\frac{12\ \text{in.}}{\text{ft.}}\right)$$

$$= 71.40\ \text{k-in.}$$

*Design for Ultimate Moment Capacity*

$$\phi M_n = \phi\left[A_s f_y\left(d_s - \frac{1}{2}\frac{A_s f_y}{0.85 f'_c\, b}\right)\right] \geq M_{STRENGTH\ I}$$ (5.5.4.2.1)

Where:

$\phi$ = Assume 0.9, then check assumption during Limits of Reinforcement check

$f_s$ = Assume 60 ksi, if $c / d_s < 0.6$ then assumption is valid (5.7.2.1)

$f'_c$ = 3.5 ksi

$d_s$ = 8 in. slab – 1.0 in. cover – 0.5 × 0.625 in. bar

= 6.6875 in.

b = 12 in.

$\phi M_n$= 121.20 k-in.

Solving for $A_s$ gives $A_s$ = 0.35 in.$^2$. Try #5 bars @ 10 in. center-to-center spacing, $A_s$ = 0.37 in.$^2$

$$c = \frac{A_s f_s}{0.85\beta_1 f'_c\, b}$$

Where:

$f_s$ = assumed to be 60 ksi

$\beta_1$ = 0.85 (5.7.2.2)

$$c = \frac{(0.37\text{ in.})(60\text{ ksi})}{(0.85)(0.85)(3.5\text{ ksi})(12\text{ in.})}$$

= 0.733 in.

$d_s$ = 6.6875 in.

$$\frac{c}{d_s} = \frac{0.733\text{ in.}}{6.6875\text{ in.}} = 0.12 < 0.6$$ $\therefore$ Assumption of $f_s = f_y$ = 60 ksi is valid.

*Check Control of Cracking* (5.7.3.4)

$$s \le \frac{700\gamma_e}{\beta_s f_s} - 2d_c \quad (5.7.3.4\text{-}1)$$

Where:

$$\beta_s = 1 + \frac{d_c}{0.7(h - d_c)}$$

Where:

$d_c$ = 1.0 in. clear + 0.5 × 0.625 in. bar

$= 1.3125 \text{ in.}$

$h = 8 \text{ in.}$

$\therefore\ \beta_s = 1.28$

$\gamma_e = 0.75$

$$f_s = \frac{M_{SERVICE\ I}}{A_s j d_s}$$

Where:

$A_s = 0.37 \text{ in.}^2$

$$j = 1 - \frac{k}{3}$$

$$k = \sqrt{(\rho n)^2 + 2\rho n} - \rho n$$

$$\rho = \frac{A_s}{bd_s} = \frac{0.37 \text{ in.}^2}{(12 \text{ in.})(6.6875 \text{ in.})} = 0.00461$$

$n = 9$

$k = 0.250$

$\therefore\ j = 0.917$

$$f_s = \frac{71.40 \text{ k} - \text{in.}}{(0.37 \text{ in.}^2)(0.917)(6.6875 \text{ in.})} = 31.5 \text{ ksi}$$

$$s \le \frac{(700)(0.75)}{(1.28)(31.5)} \text{in.} - 2(1.3125 \text{ in.}) = 10.40 \text{ in.}$$

10 in. < 10.4 in. O.K.

#5 bars @ 10 in. center-to-center spacing are adequate for crack control.

## *Check Limits of Reinforcement*

<u>Check Maximum Reinforcement</u> (5.7.3.3.1)

$$\varepsilon_t = \frac{0.003(d_t - c)}{c} \qquad \text{(C5.7.2.1-1)}$$

Where:

$c = 0.733 \text{ in.}$

$d_t = d_s = 6.6875$ in.

$$\varepsilon_t = \frac{0.003(6.6875\text{ in.} - 0.733\text{ in.})}{0.733\text{ in.}} = 0.024$$

$0.024 > 0.005$, $\therefore$ no reduction in resistance factors is required and Ultimate Moment Capacity computations are valid.

<u>Check Minimum Reinforcement</u> (5.7.3.3.2)

$M_r > M_{cr}$

$M_{cr} = \gamma_3 \gamma_1 S f_r$ (k-in.) (Eq. 5.7.3.3.2-1)

Where:

$$S = \frac{1}{6}bh^2 = \frac{1}{6}(12\text{ in.})(8\text{ in.})^2 = 128\text{ in.}^3$$

$$f_r = 0.24\sqrt{f'_c} = 0.24\sqrt{3.5\text{ ksi}} = 0.449\text{ ksi}$$ (5.4.2.6)

$\gamma_3$ = 0.75 for A706, Grade 60 reinforcement

$\gamma_1$ = 1.6 for non-segmentally constructed bridges

$M_{cr}$ $=0.75(1.6)(128\text{ in.}^3)(0.449\text{ ksi}) = 69.0$ k-in.

$$M_r = \phi M_n = (0.9)\left[(0.37\text{ in.}^2)(60\text{ ksi})\left(6.6875\text{ in.} - \frac{1}{2}\frac{(0.37\text{ in.}^2)(60\text{ ksi})}{0.85(3.5\text{ ksi})(12\text{ in.})}\right)\right]$$

$= 127.40$ k-in.

127.40 k-in. > 69.0 k-in. O.K.

**LRFD Deck Slab Design Example (continued): 7 ft. Beam Spacing, Negative Moment Reinforcement**

*Design Stresses*

As for positive moment, the design stresses are as follows:

$f_y$ = 60 ksi
$f'_c$ = 3.5 ksi

*Design Thickness*

As for positive moment, standard eight inch slab thickness.

*Determine Maximum Factored Loading*

Unfactored Loads and Moments:

$$w_{DC1} = \left(\frac{0.150\text{ k}}{\text{ft.}^3}\right)(0.667\text{ ft.})(1\text{ ft.}) = 0.100\frac{\text{k}}{\text{ft.}}$$

$$w_{DW} = \left(\frac{0.050\text{ k}}{\text{ft.}^2}\right)(1\text{ ft.}) = 0.050\frac{\text{k}}{\text{ft.}}$$

Assuming steel girders with twelve inch top flange widths, the span length shall be reduced by six inches as shown in Bridge Manual Figure 3.2.1-2 for the negative moment region.

$$M_{DC1} = \frac{1}{10}\left(0.100\frac{\text{k}}{\text{ft.}}\right)(6.5\text{ ft.})^2 = 0.423\text{ k-ft.}$$

$$M_{DW} = \frac{1}{10}\left(0.050\frac{\text{k}}{\text{ft.}}\right)(6.5\text{ ft.})^2 = 0.211\text{ k-ft.}$$

$M_{LL+IM}$ = 5.17 k-ft.

Note: This moment corresponds to the value found in Appendix A4 for a section taken three inches from the centerline of beam, for a seven foot beam spacing.

Factored Moments (Table 3.4.1-1)

$$M_{STRENGTH\ I} = \eta_i[1.25M_{DC1} + 1.5M_{DW} + 1.75M_{LL+IM}]$$

$$= 1.00[1.25(0.423 \text{ k-ft.}) + 1.5(0.211 \text{ k-ft.}) + 1.75(5.17 \text{ k-ft.})]$$

$$= 9.89 \text{ k-ft}\left(\frac{12 \text{ in.}}{\text{ft.}}\right)$$

$$= 118.68 \text{ k-in.}$$

$$M_{SERVICE\ I} = \eta_i[1.0M_{DC1} + 1.0M_{DW} + 1.0M_{LL+IM}]$$

$$= 1.00[1.0(0.423 \text{ k-ft.}) + 1.0(0.211 \text{ k-ft.}) + 1.0(5.17 \text{ k-ft.})]$$

$$= 5.80 \text{ k-ft.}\left(\frac{12 \text{ in.}}{\text{ft.}}\right)$$

$$= 69.60 \text{ k-in.}$$

*Design for Ultimate Moment Capacity*

$$\phi M_n = \phi\left[A_s f_y\left(d_s - \frac{1}{2}\frac{A_s f_y}{0.85f'_c\, b}\right)\right] \geq M_{STRENGTH\ I} \qquad (5.5.4.2.1)$$

Where:

$\phi$ = Assume 0.9, then check assumption during Limits of Reinforcement check

$f_s$ = Assume 60 ksi, if $c / d_s < 0.6$ then assumption is valid (5.7.2.1)

$f'_c$ = 3.5 ksi

$d_s$ = 8 in. slab – (2.25 + 0.25) in. cover – 0.5 $\times$ 0.625 in. bar

= 5.1875 in.

b = 12 in.

$\phi M_n$ = 118.68 k-in.

Solving for $A_s$ gives $A_s$ = 0.46 in.$^2$. Try #5 bars @ 7 in. center-to-center spacing,

$A_s$ = 0.53 in.$^2$

$$c = \frac{A_s f_s}{0.85\beta_1 f'_c\, b}$$

Where:

$f_s$ = assumed to be 60 ksi

$\beta_1$ = 0.85 (5.7.2.2)

$$c = \frac{(0.53\text{ in.})(60\text{ ksi})}{(0.85)(0.85)(3.5\text{ ksi})(12\text{ in.})}$$

= 1.05 in.

$d_s$ = 5.1875 in.

$$\frac{c}{d_s} = \frac{1.05\text{ in.}}{5.1875\text{ in.}} = 0.20 < 0.6$$ $\therefore$ Assumption of $f_s = f_y$ = 60 ksi is valid.

*Check Control of Cracking* (5.7.3.4)

$$s \le \frac{700\gamma_e}{\beta_s f_s} - 2d_c \qquad (5.7.3.4\text{-}1)$$

Where:

$$\beta_s = 1 + \frac{d_c}{0.7(h - d_c)}$$

Where:

$d_c$ = (2.25 + 0.25) in. clear + 0.5 $\times$ 0.625 in. bar

= 2.8125 in.

h = 8 in.

$\therefore\ \beta_s$ = 1.77

$\gamma_e$ = 0.75

$$f_s = \frac{M_{SERVICE\ I}}{A_s j d_s}$$

Where:

$A_s$ = 0.53 in.$^2$

$j = 1 - \frac{k}{3}$

$k = \sqrt{(\rho n)^2 + 2\rho n} - \rho n$

$\rho = \frac{A_s}{bd_s} = \frac{0.53 \text{ in.}^2}{(12 \text{ in.})(5.1875 \text{ in.})} = 0.00851$

n = 9

k = 0.322

$\therefore$ j = 0.893

$$f_s = \frac{69.60 \text{ k} - \text{in.}}{(0.53 \text{ in.}^2)(0.893)(5.1875 \text{ in.})} = 28.3 \text{ ksi}$$

$$s \leq \frac{(700)(0.75)}{(1.77)(28.3)} \text{in.} - 2(2.8125 \text{ in.}) = 4.86 \text{ in.}$$

7 in. > 4.86 in. N.G.

#5 bars @ 7 in. center-to-center spacing are not adequate for crack control.

Try #5 bars @ 6 in. center-to-center spacing, $A_s$ = 0.62 in.$^2$

$\rho$ = 0.00996

k = 0.343

j = 0.886

$$f_s = \frac{69.60 \text{ k} - \text{in.}}{(0.62 \text{ in.}^2)(0.886)(5.1875 \text{ in.})} = 24.4 \text{ ksi}$$

$$s \leq \frac{(700)(0.75)}{(1.77)(24.4)} \text{in.} - 2(2.8125 \text{ in.}) = 6.53 \text{ in.}$$

6 in. < 6.53 in. O.K.

#5 bars @ 6 in. center-to-center spacing are adequate for crack control. By inspection, c / $d_s$ is still less than 0.6, and the Ultimate Moment Capacity computation is still valid.

*Check Limits of Reinforcement*

<u>Check Maximum Reinforcement</u> (5.7.3.3.1)

$$\varepsilon_t = \frac{0.003(d_t - c)}{c} \qquad \text{(C5.7.2.1-1)}$$

Where:

$$c = \frac{(0.62\text{ in.})(60\text{ ksi})}{(0.85)(0.85)(3.5\text{ ksi})(12\text{ in.})}$$

$$= 1.22\text{ in.}$$

$$d_t = d_s = 5.1875\text{ in.}$$

$$\varepsilon_t = \frac{0.003(5.1875\text{ in.} - 1.22\text{ in.})}{1.22\text{ in.}} = 0.024$$

0.010 > 0.005, $\therefore$ no reduction in resistance factors is required and Ultimate Moment Capacity computations are valid.

<u>Check Minimum Reinforcement</u> (5.7.3.3.2)

$M_r > M_{cr}$

$M_{cr} = \gamma_3 \gamma_1 S f_r$ (k-in.) (Eq. 5.7.3.3.2-1)

Where:

$$S = \frac{1}{6}bh^2 = \frac{1}{6}(12\text{ in.})(8\text{ in.})^2 = 128\text{ in.}^3$$

$$f_r = 0.24\sqrt{f'_c} = 0.24\sqrt{3.5\text{ ksi}} = 0.449\text{ ksi} \qquad (5.4.2.6)$$

$\gamma_3$ = 0.75 for A706, Grade 60 reinforcement

$\gamma_1$ = 1.6 for non-segmentally constructed bridges

$M_{cr}$ $=0.75(1.6)(128\text{ in.}^3)(0.449\text{ ksi}) = 69.0\text{ k-in.}$

$$\phi M_n = (0.9)\left[(0.62\text{ in.}^2)(60\text{ ksi})\left(5.1875\text{ in.} - \frac{1}{2}\frac{(0.62\text{ in.}^2)(60\text{ ksi})}{0.85(3.5\text{ ksi})(12\text{ in.})}\right)\right]$$

$= 156.23\text{ k-in.}$

$156.23\text{ k-in.} > 69.0\text{ k-in.}$ O.K.

Summary:

Use #5 bars @ 10 in. center-to-center spacing for positive moment reinforcement.

Use #5 bars @ 6 in. center-to-center spacing for negative moment reinforcement.